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CALCULATING THE ICE DISCHARGE FRONT FOR DAMS (RASCHET IEDOSBROS--ETC(U)  
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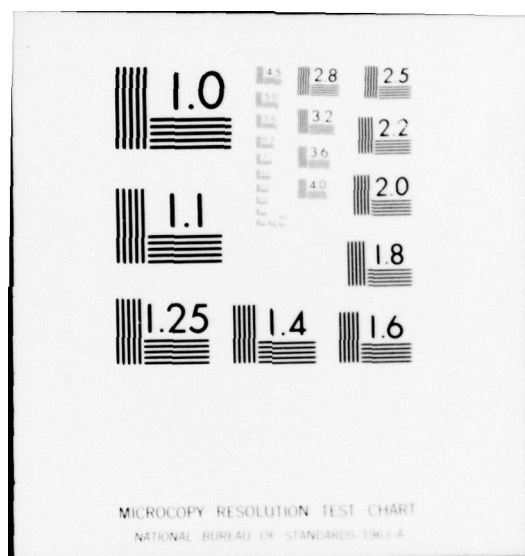
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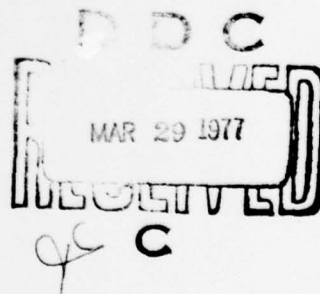
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# CALCULATING THE ICE DISCHARGE FRONT FOR DAMS

R.R. Chugayev

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Calculating the length of the so-called ice discharge front for dams is of great practical significance. The question of this type of calculation, at the basis of which is the concept of the ice discharge capacity of the stream cross section,<sup>1</sup> is discussed in what follows.

1. The ice discharge capacity,  $\Lambda$ , of a given cross section should be called the volume of ice that can pass through the given cross section in unit time

$$\Lambda = B \cdot \delta \cdot v_0 = B \cdot \delta \cdot \beta \cdot v, \quad (1)$$

where

$B$  is the upstream width of the cross section;

$\delta$  is the thickness of the ice that can move (float) through the given cross section in one, two, or several layers (taking up the entire width of the cross section);

$v_0$  is mean surface speed

$$v_0 = \beta \cdot v, \quad (2)$$

where

$v$  is the mean speed for the cross section under consideration;

$\beta$  is the factor for converting from mean speed,  $v$ , to mean surface speed,  $v_0$ , with  $\beta$  usually taken as equal to

$$\beta = 1.20 \text{ to } 1.25.$$

Eq. (1) can be reduced to the following for a rectangular bed

$$\Lambda = B \cdot \delta \cdot \beta \frac{Q}{B(h - 0.3 \cdot \delta)} = \beta \cdot \delta \frac{Q}{h - 0.3 \cdot \delta} \approx \beta \cdot \delta \frac{Q}{h}, \quad (4)$$

<sup>1</sup>This question was briefed for the first time in references [1, 2].



where

$Q$  is the water discharge;

$h$  is the depth of the stream;

$0.9 \cdot \delta$  is the settling of the ice, the magnitude of which sometimes can be ignored when compared to depth  $h$ , considering that

$$h \approx h', \quad (5)$$

where

$h'$  is the distance between the lower surface of the ice field and the bottom of the bed.

We can obtain a relationship analogous to the formula at (4) for a parabolic bed

$$\tilde{\Lambda} = \frac{3}{2} \beta \cdot \delta \cdot \frac{Q}{h - 0.9 \cdot \delta} \approx \frac{3}{2} \beta \cdot \delta \cdot \frac{Q}{h}. \quad (6)$$

Needless to say, the magnitude  $\Lambda$  can be discussed as applicable to a stream that is not covered by ice (conceivably yielding some other  $\delta$  magnitude).

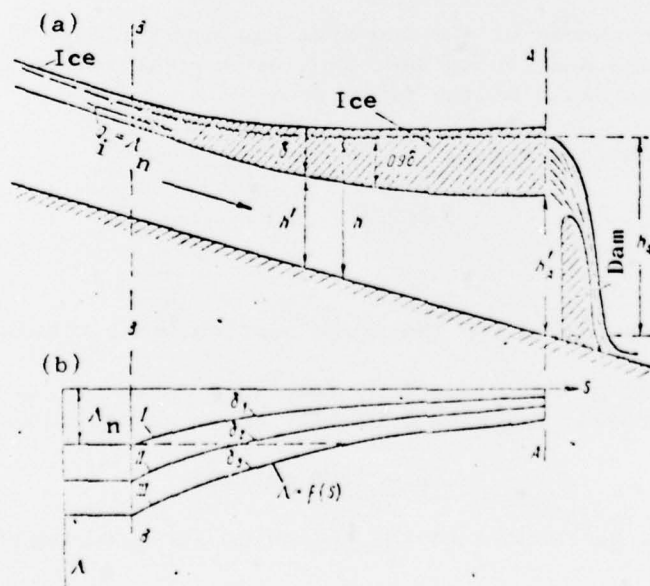


Figure 1.

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Consideration of the dependencies at (4) and (6) will show that the ice discharge capacity of a cross section in the case of conventional beds (a) will in no way depend on the width,  $B$ , of the upstream cross section, (b) is directly proportional to the ice thickness,  $\delta$ , and (c) is inversely proportional to the depth of the stream,  $h$  (strictly speaking, inversely proportional to  $h'$ ).

With these conditions in mind, let us consider the longitudinal section of the reach of the river before reaching the dam, where the reservoir can be formed (Figure 1a). Let us assume  $Q$  to be constant (in terms of time, and along the flow).

$$Q = \text{constant}. \quad (7)$$

In this case, and in accordance with the dependencies at (4) and (6),  $\Lambda$  should decrease downstream, for a given  $\delta = \text{constant}$  because depth  $h$  increases as the flow approaches the dam. A minimum,  $\Lambda_{\min}$ , should occur in the vicinity of section A-A, directly in front of the dam. Obviously,

$$\Lambda_{\min} = \beta \cdot \delta \cdot \frac{Q}{h_A} \approx \beta \cdot \delta \cdot \frac{Q}{h_A}, \quad (8)$$

where

$h_A'$  and  $h_A$  are depths equated to section A-A.

With this in mind, we have, in Figure 1b, shown the curves for

$$\Lambda = f(s) \quad (9)$$

for different  $\delta$  (see curves I, II, III..., corresponding to ice thicknesses  $\delta_1, \delta_2, \delta_3 \dots$ ). The coordinate  $s$  is shown on the drawing.

Let us take it that at the initial moment of the debacle the water course, which is in its natural state (located to the left of the vertical B-B in Figure 1a), and the reservoir (located between sections A-A and B-B), are covered by a single layer of ice, the thickness of which is  $\delta_1 = \text{constant}$ . At the same time, at the initial moment of the debacle the discharge of ice,  $Q_i$  (that is, the volume of ice passing through the given cross section in unit time), for section B-B will be equal to the ice discharge capacity of the river when the latter is in its natural state (located to the left of section B-B)

$$Q_i = \Lambda_n. \quad (10)$$

Further on downstream, in connection with the reduction in  $\Lambda$  ( $\delta = \text{constant}$ ), the thickness of the ice field should gradually increase with the passage of time because of the floes piling up on each other (because of the accumulation of ice in individual sections). If it is assumed that the inflow,  $Q_i$ , into the reservoir will last for a sufficiently long time, we should, over an extended period of time, obtain a steady-state picture of the ice



movement for which, at each section of the reservoir, there will be the following relationship

$$\Lambda = Q_i = \Lambda_n \quad (11)$$

(because of the increase in thickness,  $\delta$ , downstream). An ice field such as this, gradually thickening as it approaches the dam, is shown hatched in Figure 1a.

The result of accumulations of ice at individual sections of the stream is a rise in the free surface of the stream, accompanied by flooding of the banks. Needless to say, the debacle often can conclude before we obtain a steady-state picture of the ice movement shown in Figure 1a.

Analysis of the positions indicated for the ice movement through the reservoir will convince us that at the very beginning of the reservoir, where the backwater curve tapers, and where the depths of the downstream current begin to increase sharply, we can have an ice jam. It is important to emphasize the fact that ice jams such as these, the result of accumulations of ice (that occur in connection with the downstream reduction in  $\Lambda$ ), cannot be removed by mechanical destruction of large floes.

2. The following phenomena can occur during the debacle, as we know, and they complicate dam operating conditions.

a. Individual floes, plunging under the jam, can damage the dam in the case of an extensive, but not complete opening in rising jams.

b. Floes falling into the after bay with the water can damage the bracing of the after bay. Floes, sometimes moving at high speed, can, as they approach the dam, strike the piers and abutments and damage them. Needless to say, the ice will lose its strength as time passes (because of thawing) and no longer will be dangerous in terms of what has been said.

c. Floes moving over the crest of the dam (or over the top of the gate) in one, two, or several layers can strike the crest of the dam (or the top of the gate) and damage it. Moreover, the ice can also damage the dam's overflow surface.

d. The dam itself can be the cause of an ice jam when the layer of water flowing through the dam, or gate, is relatively shallow (when individual floes can come to rest on the crest of the dam, or gate, and stick in place), as well as when the openings are relatively narrow (when the floes can stick in these openings to form an "arch," as it were, for example, with its abutments resting in a pier). The result is that the ice can clog the approach bed, as well as the openings in the dam, reducing the discharge capacity of the approach bed and the openings in the dam and raising the water level in the head race above the permissible level. This type of jam can be referred to as a "forced jam." It is physically different from the "accumulation jam" illustrated in paragraph 1. A "forced jam" can be destroyed by different kinds of mechanical means, of course (explosives,

and the like, for example), and it is herein that the "forced jam" can be distinguished from the "accumulation jam."<sup>2</sup>

e. The depth of the water,  $h$ , at the crest of the spillway always is less than the depth,  $h_A$ , in the upper race in front of the dam, so it can be taken that when  $Q = \text{constant}$  (along the ice discharge path), the ice discharge capacity of the openings in the dam,  $\Lambda_o$ , always will be greater than the ice discharge capacity  $\Lambda_A$  (section A-A, Figure 1a). Thus, in the majority of cases there are no grounds for fearing an accumulation of ice directly in front of the dam when  $Q = \text{constant}$ . But the condition can arise when only a part of the discharge,  $Q_A$  (section A-A) will enter the ice discharge opening, installed especially for passing the ice. The other part of the discharge of water,  $Q_A$ , will, for example flow into the hydroelectric station, or into some other submerged (bottom) opening. Given these conditions, it can be shown that

$$\Lambda_o < \Lambda_A \quad (12)$$

and the ice will pile up right in front of the dam, forming an "accumulation jam" at that point, reducing the water discharge capacity of the intake channel to the dam and causing an unacceptable rise in the water level in the headrace.

The magnitude of the head,  $H$ , at the crest of the ice discharge openings (completely open) should be stipulated in order to avoid this situation, observing the condition

$$H \leq H_{\max} \quad (13)$$

where

$$H_{\max} \approx \frac{Q_o}{Q_A} \cdot k \cdot h_A, \quad (14)$$

where

$Q_o$  is the discharge of water passing through the ice discharge opening;

$k$  is a coefficient that takes into consideration the difficulty (slowing down) of movement of ice in the openings in the dam (A. R. Berezinskiy [3] recommends using  $k = 0.75$ ).

The validity of the dependency at (14) will be seen from what follows. In order to avoid an accumulation jam in front of the dam, the required relationship in this case is  $\Lambda = \Lambda_o$ , that is, the relationship [see (4) above]

$$3.8 \cdot \frac{Q_A}{h_A - 0.05} = 3.8 \cdot k \cdot \frac{Q_o}{H - 0.05}, \quad (15)$$

<sup>2</sup>The two types of jams indicated have not been differentiated to date, so efforts often are made to destroy "accumulation jams" with explosives, for example (which in such case are ineffective).

from whence, ignoring  $0.9 \cdot \delta$  and setting  $H = H_{\max}$ , we arrive at the approximate dependency at (14).

3. The following scheme for designing the ice discharge front for a dam can be contemplated on the basis of the premises elucidated above.

a. Individual floes (moving in one, two, or more layers piled atop each other), influenced by current, or overtaking winds, always can enter any spillway opening in the dam. The head,  $H$ , therefore should satisfy the condition

$$H \geq H_{\min}, \quad (16)$$

during the debacle for all water discharge openings in the dam.  $H_{\min}$  can be taken as approximately equal to 10% to 30% of the greatest thickness,  $\delta$ , of the ice mass in the section A-A (Figure 1a). As a practical matter,  $\delta$  can be taken here as equal to the thickness of one, or of two layers of ice in the majority of cases.

b. The only time a special ice discharge front is called for is when there is fear of the occurrence immediately in front of the dam of the "accumulation jam" elucidated above, one that will cause an unacceptable rise in the water level in the headrace, if such a front is omitted. This sort of "accumulation jam" can occur when the dam is low, when the ratio of the depth of the reservoir to the depth of the river emptying into it is small, the reservoir is short in length, the ice movement time is protracted, and a considerable part of the water is discharged from the headrace into the tailrace through working and bottom openings. In each concrete case, all of these factors must be specially analyzed in order to justify the need to build an ice discharge front, while at the same time giving due regard to the considerations cited above (in para. 1). It should be apparent, as a first approximation, that it is not mandatory to build a special ice discharge front for a dam in many cases encountered in practice; when, for example

$$(h_A : h_n) > 3 \text{ to } 4, \quad (17)$$

and it also can be taken that the ice in the reservoir, as well as the ice brought into the reservoir by the river can melt in the headrace, and hence will not be discharged through the dam into the tailrace. Only the condition at (16) need be satisfied in this case when planning conventional water discharge openings.

c. The width,  $b$ , of the individual openings in an ice discharge front, when the need for such front has been demonstrated (see the preceding subparagraph), is customarily set as follows: rivers with heavy debacle,  $b > 15$ -20 meters; rivers with exceptionally heavy debacle,  $b > 20$ -25 meters. It must be assumed that we will not have a "forced jam" in openings with these widths. All, or some of the water discharge openings in the dam designed to handle the water should fit under the ice discharge openings. These openings should be located along the channel line for the particular

river, or at a concave bank. The crest of the dam should have the same elevation over the limits of the openings planned for ice passage and thus ensure uniform flow of water into the openings.

The overall width of the ice discharge front,  $B_i$ , should be set in terms of the head,  $H$ , as equal to

$$H = \nabla NHE - \nabla c \quad (18)$$

and in terms of the water discharge,  $Q_o$ , as equal to

$$Q_o = Q_i - Q_w - Q_b, \quad (19)$$

where

$\nabla NHE$  is the normal headwater elevation for the water;

$\nabla c$  is the height of the crest of the dam within the limits of the ice discharge openings;

$Q_w$  and  $Q_b$  are the water discharges through the working and bottom (submerged) openings;

$Q_i$  is the discharge of water into the river during the debacle.

Knowing  $H$  and  $Q_o$ , and using the conventional water discharge formula, one can readily find the unknown  $B_i$ . It is necessary, when making this calculation, to observe the condition

$$H_{\min} \leq H \leq H_{\max}, \quad (20)$$

where

$H_{\max}$  can be found by using the formula at (14), assuming in that formula that  $Q_A = Q_i$ .

If the result of the calculation made in this manner is an  $H_{\max}$  less than  $H_{\min}$ ,  $Q_o$  must be increased at the expense of  $Q_w$  and  $Q_b$ , until the relationship  $H_{\max} \geq H_{\min}$  is reached.

There are occasions when an additional condition is imposed so that possibly less water is discharged with the ice into the tailrace. In this case  $\nabla c$  in Eq. (18) should be taken such that  $H$ , calculated by using Eq. (18) turns out to equal  $H_{\min}$  ( $H = H_{\min}$ ).

If the reservoir is in use at the onset of the debacle, no ice can be discharged through the dam until such time as the headwater level rises to the NHE. Special ice discharge openings in a dam usually need not be considered when the debacle ends during the period of time indicated, naturally. Should this not be the case, however, an ice discharge front must be planned as indicated above.



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